SEISMIC DESIGN OF JOINTED PRECAST CONCRETE WALL SYSTEMS

Sri Sritharan¹ and Sriram Aaleti²

ABSTRACT

A jointed precast concrete wall system for seismic region was introduced in Phase III of the PRESSS (PREcast Seismic Structural System) program in the United States. This wall system utilizes two or more individual precast walls that are connected to the footings using unbonded post-tensioning at the center of each wall. A link between the precast walls is established in the horizontal direction using energy dissipation elements placed along the vertical joints between the walls. Following excellent seismic performance of a wall system utilizing this concept in the PRESSS test building, a design methodology was proposed. Following identification of several shortcomings in the design methodology, this paper presents an improved methodology for seismic design of jointed precast concrete wall systems.

Keywords: precast, concrete, seismic design, wall, unbonded, post-tensioning, self-centering

INTRODUCTION

PREcast Seismic Structural Systems (PRESSS) is a research program initiated in 1991 by the US and Japan as part of research of the U.J.N.R. (U.S.-Japan Cooperative Program in Natural Resources) Panel on Wind and Seismic Effects. The US part of the PRESSS research consisted of three phases. The first phase of PRESSS focused on concept development, connection classification and modeling, analytical platform development, preliminary design recommendations, and research coordination. In Phase II, emphasis was placed on the development of ductile-connection for precast structural systems through experimental and analytical studies and development of seismic design procedures for precast buildings in various seismic regions. Integrating the components of experimental and analytical research developed in the first two phases, PRESSS Phase III was built around seismic testing of a 60% scale five-story precast concrete building. Four seismic precast frame systems and one jointed precast wall system, along with two precast floor systems, were included in the test building. The seismic frames provided the lateral load resistance in one direction of the test building while the jointed wall system was primarily responsible for lateral resistance in the orthogonal direction.

The jointed wall in the PRESSS test building consisted of two individual precast walls that were connected to the footings using unbonded post-tensioning at the center of each wall (see Fig. 1). A link between the precast walls in the horizontal direction was established using stainless steel UFP (U-shaped Flexural Plate) connectors located along a vertical joint between the walls (Figs. 1 and 2). In addition to contributing to strength, the UFP connectors served as a passive energy dissipation device, in which hysteric damping was attained by flexural yielding of the U-plates. Excellent response of the jointed wall system was observed when the building was subjected to a series of seismic tests in the wall direction (Priestley et al. 1999; also see Fig. 3). The observed damage was limited to minor

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spalling of cover concrete at the wall toes even at drifts greater than the design-level drift. Furthermore, the wall system re-centered at the end of each segment of the earthquake loading.

This paper is devoted to seismic design of jointed precast wall systems. Application of the proposed method is demonstrated through design of three different jointed wall systems.

![Diagram of jointed precast wall system](image)

**Figure 1.** Details of the jointed two-wall system included in the PRESSS test building (1 ft = 0.3048 m; 1 inch = 25.4 mm).

![Detail of UFP connector](image)

**Figure 2.** Details of the UFP connector used in the PRESSS wall system.

![Graph of force-displacement response](image)

**Figure 2.** Details of the UFP connector used in the PRESSS wall system.
Figure 3. Seismic response of the PRESSS building in the wall direction.

BACKGROUND

At the completion of the PRESSS program, Stanton and Nakaki (2002) published a set of guidelines for the design of all five precast structural systems tested in the PRESSS building. Subsequently, the guidelines proposed for the precast jointed wall system were examined by Thomas and Sritharan (2004) using the data from the PRESSS test building. The design methodology summarized in this paper generally follows that proposed by Stanton and Nakaki (2002) and addresses the shortcomings of the methodology identified by Thomas and Sritharan. A critical issue that was not addressed in the original design method was how the design base moment determined for the wall system should be divided up between the individual precast walls. The proposed methodology not only addresses this issue, but proposes a simplification in the methodology by requiring the design of only the most critical wall. The remaining walls in the jointed system are detailed with essentially the same post-tensioning and connector details.

METHODOLGY

Design Assumptions

Consistent with the recommendations of Stanton and Nakaki (2002), the following assumptions are made for the design of jointed precast wall systems.

- The wall will undergo in-plane deformations only. Torsion and out-of-plane deformations are prevented by providing adequate out-of-plane bracing.
- All individual walls are assumed to have identical dimensions, reinforcement details, and the initial prestressing force.
- All the vertical joints contain an equal number of identical connectors, and a dependable force vs. displacement response envelope is available for the connector (e.g., see Fig. 2b).
- All walls undergo the same lateral displacement at the floor and roof levels due to the rigid floor assumption.
- The post-tensioning steel is located at the center of each wall.
- The post-tensioning steel reaches the yield strain at the design drift. The corresponding rotation at the wall base is assumed to be $\theta_{\text{design}}$, which may be taken as 2%. Alternatively, use an acceptable wall design drift to estimate a suitable value for $\theta_{\text{design}}$. 
Summary of Parametric Study

Lateral load resistance of a jointed wall system at a given drift depends on the geometry and number of walls in the system, the amount of post-tensioning steel, the number of vertical connectors, initial prestressing force, and the cyclic behavior of the vertical connector. To understand the effects of several of these design parameters on the response of jointed wall systems, a parametric study was conducted involving jointed systems consisted of two, three, and four precast walls (Aaleti 2004). All wall systems were assumed to satisfy the aforementioned assumptions. From this study, two major conclusions were drawn:

1. In a jointed two-wall system, the leading wall will provide about 2/3rd of the total lateral force resistance. However, in a jointed wall system having more than two walls, each of the intermediate walls will provide larger moment resistance than the leading or the trailing wall. The percentage contribution of an intermediate wall towards the total base moment resistance will depend on the number of walls in the jointed system.

2. As suggested by Thomas and Sritharan (2004), the post-tensioning steel in the trailing wall would first reach the yield limit state in a jointed wall system, which should dictate the initial design stress for the post-tensioning steel. However, the wall providing the largest moment resistance should be used to design the required area of the post-tensioning steel.

Design Steps

Incorporating the findings from the parametric study, a procedure was finalized for seismic design of jointed precast walls, which can be summarized using the following seven steps.

Step 1: Material Properties and Wall Dimensions

- Select the following material properties.
  Prestressing steel: Modulus of elasticity (E_p) and yield strength (f_{py}).
  Concrete: Unconfined concrete strength (f'_c), elastic modulus of concrete (E_c) which may be approximated to $47000f'_c$ (MPa), and appropriate coefficient of friction between the precast wall base and foundation ($\mu$).
  Connector: Force vs. displacement response envelope.
- Establish the wall dimensions.
  Select the total length of the wall system (L_s) or length of a single wall (L_w), wall height (H_w), wall thickness (t_w), and the number of walls (n). The height and length of the wall system can be determined from the architectural drawings or from preliminary design calculations.

When deciding the number of walls in each system, use a smallest possible value for n with a suitable L_w/H_w ratio. Stanton and Nakaki (2002) suggest that L_w/H_w should be more than 2.0 to ensure flexural dominant behavior for the wall. Consequently, if the length of each wall or the total length of the wall system is known, the other variable can be determined from Eq. 1.

$$L_w = \frac{L_s}{n} \quad (1)$$

The following guidance may be used to determine an initial value for the wall thickness:

a) Select a value in the range of h_{story}/16 to h_{story}/25, where h_{story} is the story height (Englekirk 2003);

b) Ensure that the selected wall thickness should be sufficient to limit the shear stress in the wall to the permissible limit specified in the current building standard (e.g., ACI 318-05 2005); and
c) The selected wall thickness should be sufficient to accommodate the required confinement reinforcement at the wall ends without causing any construction difficulties.

**Step 2: Required Design Moment Resistance**

Using a force-based design (FBD) or displacement-based design (DBD) procedure, arrive at the required base moment resistance for the wall system ($M_{\text{design}}$). Hence, the precast wall system should be designed such that

$$\phi M_n \geq M_{\text{design}}$$  \hspace{1cm} (2)

where $\phi$ is the flexural strength reduction factor and $M_n$ is the nominal moment capacity of the wall system at the design drift.

**Step 3: Force Resisted by the Connector**

- Assuming a vertical relative displacement between the walls to be $0.9L_w\theta_{\text{design}}$, estimate the force in the connector ($F_{\text{con}}$) at the design drift from the force-displacement envelope curve available for the connector (e.g., see Fig. 2b).
- For the wall systems described above, a symmetric lateral response is expected when they are subjected to symmetric cyclic loading. For such systems, the hysteretic energy dissipation can be correlated to equivalent viscous damping using Eq. 3.

$$\zeta_{\text{eq}} = \frac{2A_{\text{loop}}}{\pi A_{\text{rect}}}$$  \hspace{1cm} (3)

where $A_{\text{loop}}$ is the area enclosed by a symmetric hysteresis loop at the design drift and $A_{\text{rect}}$ is the area of the rectangle circumscribing the hysteresis loop.

- The number of connectors should be determined such that a desired level of equivalent damping is incorporated in the wall system. If UFP connectors, as successfully used in the PRESSS wall, are chosen, then the required number of connectors may be established from Eq. 4 to ensure that the wall system would have a desired level equivalent damping (Galusha 1999).

$$N_{\text{con}} = \frac{\pi \zeta_{eq} M_n}{1.25(n-1)F_{\text{con}} L_w}$$  \hspace{1cm} (4)

where $N_{\text{con}}$ is the number of connectors in each vertical joint between the precast walls and $\zeta_{eq}$ is the required level of equivalent viscous damping, which should be in the 15 to 20 percent range to ensure that the wall system will have both adequate damping and recentering capability.

**Step 4: Required Area of the Post-tensioning Steel**

- In a jointed wall system containing walls with equal dimensions, the design moment for the wall that would provide the largest moment resistance can be determined from Eq. 5.

$$M_{\text{design,wall}} = \Omega \frac{M_{\text{design}}}{n\phi}$$  \hspace{1cm} (5a)
\[
\Omega = 1 + \frac{\alpha \phi N_{\text{con}} F_{\text{con}} L_w}{M_{\text{design}}}
\]

where \(\Omega\) is the moment contribution factor and \(\alpha\) is a constant. When \(n = 2\), \(\alpha = 0.9\) and \(M_{\text{design,wall}}\) will correspond to the moment demand in the leading wall (i.e., \(M_{\text{design,lead}}\)). When \(n \geq 3\), \(\alpha = 1.03\) and the \(M_{\text{design,wall}}\) will correspond to the moment in an intermediate wall (i.e., \(M_{\text{design,inter}}\)). The suitable values for \(\alpha\) were established during the parametric study that was noted previously.

- If the jointed system contains only two walls (i.e., \(n = 2\)), design the area of the post-tensioning steel, \(A_p\), using Eq. 6, which uses the moment equilibrium of forces acting on the base of the leading wall (see Fig. 4).

\[
M_{\text{design,lead}} = \left(P_D + 0.95F_{\text{py}} A_p\right) * \left(\frac{L_w}{2} - \frac{P_D + 0.95F_{\text{py}} A_p + N_{\text{con}} F_{\text{con}}}{2 * (1.6f'_c) t_w}\right)
\]

where \(P_D\), the summation of the wall self weight and superimposed live load, is equated to \((\gamma_c L_w t_w H_w + W_{\text{floor}} L_w)\), \(\gamma_c\) is the unit weight of concrete, \(W_{\text{floor}}\) is the distributed superimposed live load at the base of wall from all floors, \(0.95F_{\text{py}}\) represents the expected stress in the post-tensioning steel in the critical wall at the design drift, and \(1.6f'_c\) approximates the expected confined concrete strength of the equivalent rectangular stress block. Eq. 6 will lead to a quadratic equation in \(A_p\) and the small positive root should be used as the design value for \(A_p\).

- Similarly for a multi-wall system with \(n \geq 3\) (Fig. 4), the required area of the post-tensioning steel is established using the moment equilibrium of the forces acting at the base of an intermediate wall, as detailed in Eq. 7.

\[
M_{\text{design,inter}} = \left(P_D + 0.95F_{\text{py}} A_p\right) * \left(\frac{L_w}{2} - \frac{P_D + 0.95F_{\text{py}} A_p + N_{\text{con}} F_{\text{con}}}{2 * (1.6f'_c) t_w}\right) + N_{\text{con}} F_{\text{con}} L_w
\]

Figure 4. Forces acting on a jointed three-wall system at base rotation \(\theta\) (\(C=\) resultant compressive force and \(T = P_D + \) force in the prestressing tendon)

Similarly for a multi-wall system with \(n \geq 3\) (Fig. 4), the required area of the post-tensioning steel is established using the moment equilibrium of the forces acting at the base of an intermediate wall, as detailed in Eq. 7.
The connector forces acting on both sides of an intermediate wall will not be equal. However, they are assumed to be the same in Eq. 7 to simplify the design procedure. As with the leading wall design in Eq. 6, Eq. 7 will lead to a quadratic equation in \( A_p \) and the small positive root should be taken as the design value for \( A_p \).

**Step 5: Design the Initial Stress for the Post-tensioning Steel**

- Using Eq. 8, estimate the neutral axis depth at the base of the trailing wall at the design drift.

\[
c_{\text{design, trail}} = \frac{P_D + F_{py} A_p - N_{con} F_{con}}{0.85 \times (1.6 f'_c) t_w}
\]

(8)

- Assuming that the post-tensioning steel reaches the yield limit state in the trailing wall at the design drift, the initial stress in the post-tensioning steel is established from Eq. 9.

\[
F_{pi} = F_{py} - \frac{(0.5 L_w - c_{\text{design, trail}}) \theta_{\text{design}} F_p}{H_w}
\]

(9)

**Step 6: Estimate the Moment Capacity**

The connector details, the area of the post-tensioning steel and the initial prestress designed in the previous steps are recommended for use in all walls in the jointed system, which would simplify the design instead of designing the walls individually as postulated by Stanton and Nakaki (2002). Using a suitable analysis procedure (e.g., Aaleti 2004; Thomas and Sritharan 2002), estimate the total base moment resistance of the jointed wall system and ensure that Eq. 2 is satisfied. Based on the examples investigated to date by the authors, the proposed design method appears to adequately satisfy Eq. 2 and no further iteration was found to be necessary. However, if Eq. 2 is not satisfied in a design problem, it is recommended that wall dimensions be altered in order to improve the design.

**Step 7: Design of Confinement Reinforcement**

With the jointed connection between the wall and foundation, strain concentrations are expected at the compressive regions of the wall toes. A realistic maximum strain demand has not been successfully established from experiments or analyses. However, using the data from the PRESSS test building and identifying that the leading wall would experience the maximum resultant compressive force at the base, Eq. 10 has been suggested for estimating the maximum concrete strain demand in the compressive regions of the wall toes (Thomas and Sritharan 2004).

\[
\varepsilon_{\text{conc}} = c_{\text{max, lead}} \left( \frac{M_{\text{max, lead}}}{E_c I_{\text{gross}}} + \frac{\theta_{\text{max}}}{0.06 H_w} \right)
\]

(10)

where \( M_{\text{max, lead}} \) is the base moment resistance of the leading wall at the maximum expected drift, the corresponding base rotation is \( \theta_{\text{max}} \), which may be taken as \( 1.5 \theta_{\text{design}} \), \( I_{\text{gross}} \) is the gross moment of inertia of the wall and is equal to \( t_w L_w^3 / 12 \), and \( c_{\text{max, lead}} \) is the neutral axis depth at the base of the leading wall at \( \theta_{\text{max}} \). The value of \( c_{\text{max, lead}} \) may be established as part of the analysis of the wall system in Step 6. Following an estimate for \( \varepsilon_{\text{conc}} \) from Eq. 10, quantify the required amount of confinement reinforcement in the wall toes using an appropriate confinement model. If the model proposed by Mander et al. (1988) is selected, then Eq. 11 will be used to determine the required amount of confinement reinforcement.
\[ \rho_s = \frac{(\varepsilon_{\text{conc}} - 0.004) f'_{cc}}{1.4 f_y h \varepsilon_{su}} \]  

where \( \rho_s \) is the volumetric ratio of the required confinement steel, \( f_y \) and \( \varepsilon_{su} \) are, respectively, the yield strength and ultimate strain capacity of the confinement reinforcement, and \( f'_{cc} \) is the ultimate strength of the confined concrete. Since \( f'_{cc} \) is dependent on the value of \( \rho_s \), an iterative approach would be necessary to solve Eq. 11. For the first step in the iteration, \( f'_{cc} \) may be approximated to 1.6\( f'_{su} \). In addition to satisfying the requirement of Eq. 11, it is suggested that the selected confinement reinforcement should satisfy all seismic design provisions prescribed in the current building standard for the design of transverse reinforcement in the plastic hinge region of a concrete wall.

Finally, using the friction coefficient of \( \mu \), the shear resistance at the base of the wall should be ensured using a shear friction mechanism. If an interface material such as grout is placed between the precast walls and foundation, this should be reflected in the value of \( \mu \). Since the stress in the post-tensioning steel and the connector force increase with drift, it will be necessary to perform this check at both \( \theta_{\text{design}} \) and \( \theta_{\text{max}} \).

**DESIGN EXAMPLES**

Using the design methodology presented above, this section presents three examples for the design of precast jointed wall systems. For all examples, the design base moment is assumed to be 8562 kN-m. Furthermore, the length of each wall, total number of connectors, and materials properties are also assumed to be the same for all wall systems. The number of walls in each system is varied such that they represent two-, three- and four-wall systems. The properties selected for the examples are those of the PRESSS wall system, and thus the two-wall design example is comparable to the wall system used in the PRESSS test building. The three- and four-wall system examples are not expected to be very efficient design, but they serve adequately to validate the design methodology.

Table 1 summarizes the design of all three wall systems by presenting results obtained at various design steps. In all examples, the flexural strength reduction factor of 1 is purposely used to illustrate the accuracy of the design method. This can be seen clearly in Fig. 5, which compares the design moment with the nominal moment capacity established at the design drift for the critical wall and the wall system for each example. In all cases, it is seen that the calculated nominal capacity is slightly greater than the moment demand.

![Comparison of the design moment with the nominal moment capacity for the three examples.](image-url)
### Table 1. Summary of three examples showing the design of jointed precast wall systems

<table>
<thead>
<tr>
<th>Design Steps</th>
<th>Two-wall system</th>
<th>Three-wall system</th>
<th>Four-wall system</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Assume the following parameters:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\theta_{\text{design}}$ at the design drift</td>
<td>2%</td>
<td>2%</td>
<td>2%</td>
</tr>
<tr>
<td>$\theta_{\text{max}}$ at the expected maximum drift</td>
<td>3%</td>
<td>3%</td>
<td>3%</td>
</tr>
<tr>
<td><strong>Step 1:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Material Properties:</strong></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Young’s modulus for PT bar ($E_p$)</td>
<td>$1.91 \times 10^5$ MPa</td>
<td>$1.91 \times 10^5$ MPa</td>
<td>$1.91 \times 10^5$ MPa</td>
</tr>
<tr>
<td>Yield strength of PT bar ($f_{py}$)</td>
<td>965 MPa</td>
<td>965 MPa</td>
<td>965 MPa</td>
</tr>
<tr>
<td>Concrete strength ($f'c$)</td>
<td>41.4 MPa</td>
<td>41.4 MPa</td>
<td>41.4 MPa</td>
</tr>
<tr>
<td>Elastic modulus of concrete ($E_c$)</td>
<td>30.2 GPa</td>
<td>30.2 GPa</td>
<td>30.2 GPa</td>
</tr>
<tr>
<td>Concrete density ($\gamma_c$)</td>
<td>$2.4 \times 10^3$ kg/m$^3$</td>
<td>$2.4 \times 10^3$ kg/m$^3$</td>
<td>$2.4 \times 10^3$ kg/m$^3$</td>
</tr>
<tr>
<td>Response of connector</td>
<td>Use Fig. 2b</td>
<td>Use Fig. 2b</td>
<td>Use Fig. 2b</td>
</tr>
<tr>
<td><strong>Wall dimensions:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of a single wall ($L_w$)</td>
<td>2.74 m</td>
<td>2.74 m</td>
<td>2.74 m</td>
</tr>
<tr>
<td>Thickness of the wall ($t_w$)</td>
<td>0.2 m</td>
<td>0.2 m</td>
<td>0.2 m</td>
</tr>
<tr>
<td>Height of the wall ($H_w$)</td>
<td>11.43 m</td>
<td>11.43 m</td>
<td>11.43 m</td>
</tr>
<tr>
<td><strong>Step 2:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design moment ($M_{\text{design}}$)$^1$ (Established from DBD)</td>
<td>8562 kN-m</td>
<td>8562 kN-m</td>
<td>8562 kN-m</td>
</tr>
<tr>
<td><strong>Step 3:</strong></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>No. of connectors per joint$^2$</td>
<td>20</td>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>Force in each connector ($F_{\text{con}}$) at $\theta_{\text{design}}$</td>
<td>51.8 kN</td>
<td>51.8 kN</td>
<td>51.8 kN</td>
</tr>
<tr>
<td><strong>Step 4:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select the critical wall</td>
<td>Leading wall</td>
<td>Inter. Wall</td>
<td>Inter. Wall</td>
</tr>
<tr>
<td>Flexural strength reduction factor ($\phi$)$^3$</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Moment contribution factor ($\Omega$)</td>
<td>1.3</td>
<td>1.17</td>
<td>1.12</td>
</tr>
<tr>
<td>Design moment of the critical wall</td>
<td>5560 kN-m</td>
<td>3342 kN-m</td>
<td>2397 kN-m</td>
</tr>
<tr>
<td>Required post-tensioning steel area</td>
<td>23.2 cm$^2$</td>
<td>14.3 cm$^2$</td>
<td>9.8 cm$^2$</td>
</tr>
<tr>
<td><strong>Step 5:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Initial stress in post-tensioning steel</td>
<td>546.5 MPa</td>
<td>536.3 MPa</td>
<td>528.4 MPa</td>
</tr>
<tr>
<td><strong>Step 6:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Critical wall capacity from analysis</td>
<td>5605 kN-m</td>
<td>3423 kN-m</td>
<td>2451 kN-m</td>
</tr>
<tr>
<td>Moment capacity of joined wall system</td>
<td>8891 kN-m</td>
<td>8850 kN-m</td>
<td>8812 kN-m</td>
</tr>
<tr>
<td><strong>Step 7:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M_{\text{max}}$ of critical wall at $\theta_{\text{max}}=0.03$</td>
<td>6301 kN-m</td>
<td>3790 kN-m</td>
<td>2737 kN-m</td>
</tr>
<tr>
<td>Neutral axis depth</td>
<td>0.283 m</td>
<td>0.173 m</td>
<td>0.125 m</td>
</tr>
<tr>
<td>Required concrete strain capacity</td>
<td>0.0126</td>
<td>0.00765</td>
<td>0.0055</td>
</tr>
<tr>
<td>Required confinement steel ratio ($\rho_s$)</td>
<td>0.0081</td>
<td>0.0034</td>
<td>0.0014</td>
</tr>
<tr>
<td>Confinement steel required by code$^4$</td>
<td>0.012</td>
<td>0.012</td>
<td>0.012</td>
</tr>
</tbody>
</table>

$^1$Calculation of the design moment may be found in Sritharan et al. (2002).
$^2$The total number of connectors in each wall system was kept the same as that used in the PRESSS wall system.
$^3$The value of $\phi$ is taken as 1.0 instead of 0.9 only to demonstrate the accuracy of the design procedure.
$^4$Only the minimum confinement requirement of $0.12 \frac{f'c}{f_{yh}}$ as per ACI 318-05 (2005) is ensured.
CONCLUSIONS

A design methodology established for the seismic design of precast jointed wall systems is summarized in this paper. The wall system uses two or more precast concrete walls having equal dimensions. The walls are anchored to the footings using unbonded post-tensioning tendons in the center of each wall. Furthermore, the walls are connected horizontally using special connectors in the joints between the walls. This wall concept, which was experimentally studied in the PRESSS test building, was proved to be an excellent choice for seismic design of precast wall systems.

The design methodology presented for the jointed wall systems includes the recommendations of Stanton and Nakaki (2002), addresses the shortcomings in design of this wall system identified by Thomas and Sritharan (2004), and simplifies the design procedure wherever possible. By performing three examples, it is shown that this design methodology is simple and satisfactory. For these examples, it is also shown that the minimum code requirement for the confinement reinforcement would be adequate in the wall compression toes. Although it was assumed that the unbonded post-tensioning steel is concentrated at the center of the walls, it may be distributed symmetrically along the wall length. In such cases, most equations detailed in the design methodology can be used except for Eq. 9. The design of the initial post-tensioning force in Eq. 9 should be determined using the force in the post-tensioning steel furthest from the wall compression toe in the trailing wall in order to ensure that no yielding of the tendon would occur at the design drift. In all other equations, $A_p$ should represent the total area of the prestressing steel.

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